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Summary of Long-Term Rapid Infiltration System Studies

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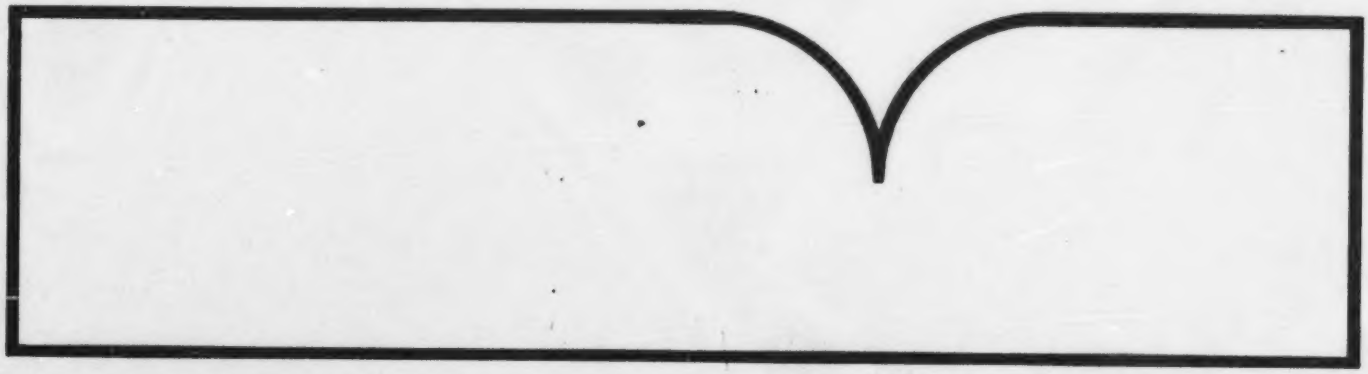

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This project was initiated with the objective of summarizing and comparing the data published in individual reports of four long-term rapid infiltration municipal wastewater systems. Evaluation of this data provides the user community with a condensed assessment of the treatment received at each of these sites from which projected treatment of planned systems can be estimated.

In addition to a summarization and evaluation of data, a hypothetical design was made for each system based on a rationale for both hydraulic loading and effluent water quality considerations. This rationale is proposed as a supplement to the design criteria presented in the Process Design Manual for Land Treatment of Municipal Wastewater, EPA-625/1-77-008, 1977.

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SUMMARY OF LONG-TERM RAPID INFILTRATION SYSTEM STUDIES

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FOREWORD

The Office of Research and Development of the Environmental Protection Agency conducts research involving the search for information about environmental problems, management techniques, and new technologies to identify, control, and eliminate the threat pollution poses to the welfare of the American people. This research is conducted through a nationwide network of research facilities.

As one of these facilities, the Robert S. Kerr Environmental Research Laboratory is responsible for the management of programs to: (a) investigate the nature, transport, fate and management of pollutants in ground water; (b) develop and demonstrate methods for treating wastewaters with soil and other natural systems; (c) develop and demonstrate pollution control technologies for irrigation return flows; (d) develop and demonstrate pollution control technologies for animal production wastes; (e) develop and demonstrate technologies to prevent, control or abate pollution from the petroleum refining and petrochemical industries; and (f) develop and demonstrate technologies to manage pollution resulting from combinations of industrial wastewaters or industrial/municipal wastewaters.

The purpose of this report is to summarize the results of detailed studies of four rapid-infiltration systems. These studies were a part of a major effort by EPA to determine the long-term effects of land treatment of municipal wastewater. For each of the four projects summarized in this report, a detailed final report has been prepared. These reports contain complete data and data evaluation. The objective of this report is to provide a summary of pertinent information which may, in many cases, be sufficient for the users. Those who are interested in the complete data from any of these projects should refer to the individual project reports. It is hoped that this summary will be useful to those who do not require this much detail.

William C Galegar

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ABSTRACT

This research project was initiated with the overall objective of summarizing and comparing the data generated from individual reports of four long-term rapid infiltration municipal wastewater systems. Evaluation of this material gives the user community a condensed assessment of the treatment received at each of these sites from which projected treatment of planned systems can be made.

During the review and summary process, the treatment efficiency of the systems was compared based on level of effluent pretreatment, hydraulic loading rate, depth of soil profile available for treatment, and variation in operational practice.

In addition to a summarization and evaluation of data, a hypothetical design was made for each system based on a rationale for hydraulic loading and effluent water quality considerations. This rationale is proposed as a supplement to the design criteria presented in the Process Design Manual for Land Treatment of Municipal Wastewaters (1). The rationale considers, in addition to hydraulic acceptability of the most restrictive layer, water quality parameters which can also be limiting. These parameters include: sodium adsorption ration (SAR), biochemical oxygen demand (BOD), suspended solids (SS), ammonia (NH_4^+), nitrate (NO_3^-), and phosphorus (P).

Testing of the design of each of these systems with measured water quality data indicated three of the systems are being operated to produce the quality of effluent predicted under present design considerations. The fourth system should be capable of improving operation by underdraining and changing management practices.

CONTENTS

Foreword iii
Abstract iv
Figures vi
Tables vii

1. Introduction. 1
2. Conclusions 3
3. Recommendations 5
4. Description of Four Existing Systems
    General 7
    Hollister, California 7
    Lake George, New York 10
    Vineland, New Jersey 11
    Milton, Wisconsin 12
5. Comparison of Applied and Renovated Wastewater 14
6. Observed Changes in Soil Chemical Properties 24
7. Rationale for Rapid infiltration Design 29

References 49

FIGURES

Number		Page
1	pH comparisons in the soil profile	25
2	Zinc concentrations in the soil profile	25
3	Copper concentrations in the soil profile	26
4	Total phosphate comparisons in the soil profile . . .	26
5	Calcium carbonate reduction in the soil profile. . . .	28
6	Organic nitrogen reduction comparisons in the soil profile	28
7	Nitrification and denitrification versus temperature .	31

TABLES

Number		Page
1	Initial Site Screening Criteria	8
2	Operating Factors for Existing Rapid Infiltration Systems	9
3	Comparison of Applied Wastewater vs. Shallow Ground Water	16
4	Comparison of Shallow Ground Water to Control Wells	18
5	Input Data Required to Evaluate Hollister Site . . .	36
6	Hollister Design Evaluation	38
7	Input Data Required to Evaluate Milton Site	39
8	Milton Design Evaluation	41
9	Input Data Required to Evaluate Lake George Site . .	42
10	Lake George Design Evaluation	44
11	Input Data Required to Evaluate Vineland Site	46
12	Vineland Design Evaluation	48

SECTION 1

INTRODUCTION

Many years of sewage farming, both in Europe and in the United States, show land treatment systems using municipal sewage for irrigation and ground water recharge can be managed so that they are viable alternatives to conventional municipal sewage treatment systems. The number of these systems is steadily growing, particularly in cities where land is available and irrigated agriculture is normally practiced. But, nevertheless, only a small percentage of the total number of municipalities use land treatment systems.

An intense effort is being made by federal agencies to develop land treatment technology and improve control practices. Through these efforts and through practical experience, three basic types of land treatment systems are now recognized. These three systems are identified as slow rate (crop irrigation), rapid infiltration (infiltration-percolation), and overland flow systems.

With the passage of the Federal Water Pollution Control Act Amendments of 1972 (Public Law 92-500), there was a new impetus to utilizing land treatment systems in the United States. In the Act, the Administrator of the Environmental Protection Agency (EPA) is directed to encourage construction of wastewater treatment systems which: (1) recycle potential pollutants through the production of agricultural, silvicultural, and aquacultural products; (2) reclaim wastewater; and (3) eliminate the discharge of pollutants. Following the passage of this legislation, the EPA adopted the policy that land treatment must be evaluated as an alternative for all wastewater treatment systems funded under its Construction Grants Program.

With the strong committment of the Federal Government to encourage the adoption of land treatment, where appropriate, planners and designers were faced with the problem of inadequate and incomplete design criteria for these systems. Furthermore, there was strong resistance, in many quarters, to the use of land treatment. This resistance came from state regulatory agencies, the engineering community, government officials at all levels, and often from the general public. Much of this resistance was generated because of questions related to health effects, lack of reliable design criteria, and long-term effects of such systems.

In order to provide answers to these questions, the EPA, during the latter part of 1975, awarded contracts to study the long-term effects of eight existing land treatment systems. The sites selected for study included

five slow rate and three rapid infiltration systems. The sites were selected to represent different preapplication treatments, different geographical locations, different soil types, and different management practices. The contractors were to collect data on numerous chemical and biological parameters and compare the properties of crops, soils, and ground water at the wastewater management sites against appropriate control sites. A basic criterion in site selection was that all study sites had to have an uninterrupted history of at least ten years of operation.

In addition to the eight systems studied under these contracts, two other systems were studied under grants, making a total of ten study sites. Of the ten systems evaluated, four were rapid-infiltration systems and six were slow rate systems. All projects have been completed and individual reports have been prepared reporting, in detail, the results of the studies.

The purpose of this report is to summarize the results of the four rapid-infiltration studies. The individual project reports contain all collected data and are quite voluminous. It was felt that a summary, such as presented in this report, would be helpful to many readers. Those who have the need for complete details of the studies, however, should refer to the individual project reports.

Subsequent to the initiation of these projects, there was published the "Process Design Manual for Land Treatment of Municipal Wastewater" (1). This manual was a cooperative effort of the U. S. Army Corps of Engineers, the U. S. Department of Agriculture, and EPA. This was the first design manual ever published for land treatment systems, and it was recognized that it was deficient in many respects because of lack of reliable data. A major benefit of the existing system studies is to provide new and expanded data for the updating and revising of the design manual. Included in this report are designs of the systems studied based on collected data with comparisons of design criteria contained in the design manual. These comparisons show where inadequate criteria exist and which areas of the manual need modification and expansion.

SECTION 2

CONCLUSIONS

After careful review and comparison of the treatment of the four rapid infiltration systems, a number of conclusions can be made.

Two of the systems were applying primary pretreated wastewater to infiltration beds and two were applying secondary pretreated wastewater. Comparison of the treatment efficiency of infiltration beds for these two levels of pretreated wastewater indicated a higher percentage of removal for selected parameters occurred in the systems with primary pretreatment. Both levels of pretreatment demonstrated the potential for producing a high quality effluent. Fecal coliform reductions clearly need improvement, particularly where shallow water tables exist.

Only one of the four systems evaluated the effects of seasonal changes on the treatability of wastewater even though all four systems were operated year-round. It was found that these systems can be successfully managed and operated through severe winters, with reasonable treatment efficiency as demonstrated in upper New York State at Lake George.

Lake George, New York, and Hollister, California, demonstrated exceptional capability for nitrogen removal through the soil treatment systems. Both of these systems were managed using well scheduled wetting and drying cycles. Apparently, the total organic carbon available for denitrification and the management utilized at Lake George and Hollister account for the effectiveness of treatment in these two systems.

Phosphorus concentration was reduced at each of the treatment sites. The concentration in the shallow ground water at all test sites was greater than in the corresponding control ground water, indicating that with the application rates and detention times used, phosphorus did migrate with the applied wastewater.

Iron appears to be leaching from the profile at all sites, possibly due to reducing conditions developed during flooding cycles. Generally, the other metals reported at the sites did not indicate significant differences in concentration between the wastewater applied and that reaching the shallow ground water. All metal concentrations, however, were within acceptable limits for drinking water quality (2).

Copper, nickel, and zinc showed significant accumulation near the surface of the soil profiles. This accumulation appears to be a process of precipitation rather than ion exchange since there was little change in ionic ratios of the cations. Phosphorus was also shown to accumulate in the soil profiles throughout the 300 centimeters (cm) [9.8 feet (ft)] monitored. Organic nitrogen accumulated at the soil surface but accumulations did not penetrate below 30 cm (1 ft) at any site. In the one system where calcium carbonate was monitored in the soil profile, significant leaching was apparent. The operation of this system was reported to have neutralized the pH of the soil profile.

The pesticides that were detected in the wastewaters were all found in the shallow ground water indicating these pesticides are mobile in the soil profile. Lindane was the only pesticide that was found significantly elevated over background ground water concentrations.

Evaluation of the treatment achieved in the four system indicates they are generally operating in conformance with current critera published in the Process Design Manual for Land Treatment of Municipal Wastewaters (1) as well as the proposed design rationale.

SECTION 3

RECOMMENDATIONS

Rapid infiltration systems have been shown to effectively treat municipal wastewater. Due to their minimal area requirements, it is recommended that this mode of wastewater treatment be given careful consideration as a viable treatment alternative.

Observations of all four sites indicated fecal coliforms were leached to the ground water. Additional research is required to delineate the soil's ability to remove fecal coliform. This research needs to determine the effects of soil temperature, detention time, and soil texture on the soil's ability to remove fecal coliforms.

Some treatment sites showed less than optimum nitrogen removal. Careful consideration should be given in evaluating the drainage requirements to assure adequate oxidation of the wastewater. Nitrogen problems are also closely related to operational practices. Loading at more than 6 cm/hour (hr) (2.4 inches (in)/hr) or loading cycles with less than three days of drying tend to reduce treatment effectiveness. When plans for operation beyond these limits are being developed, pilot scale testing may be needed to demonstrate treatment capability at specific sites.

Metal ions showed potential mobility through the soil profile. Additional study is recommended to develop a design rationale capable of addressing this potential problem. It should be noted that municipal wastewater generally meets drinking water standards for metals, and problems are not anticipated except where significant industrial input is included in the waste stream.

The total organic carbon in the applied wastewater is significantly reduced through land treatment. It was observed, however, that some organopesticides did move undegraded through the soil profile. Study is needed to determine which toxic organics will not degrade and which of these show a potential for mobility. Pretreatment may be required for those organic compounds which might contaminate the ground water.

It is recommended that the revised rationale of rapid infiltration system design presented in this report be tentatively adopted and tested for use in updating the Process Design Manual for Land Treatment of Municipal Wastewaters (1). This rationale considers, in addition to loading design based on the present parameters of hydraulic acceptability of the most restrictive layer,

loading design based on water quality parameters which might be even more limiting. These parameters include ammonium (NH_4^+), nitrate (NO_3^-), and phosphorus (P).

SECTION 4

DESCRIPTION OF FOUR EXISTING SYSTEMS

GENERAL

The four existing rapid infiltration systems evaluated and compared include Hollister, California (3); Lake George, New York (4); Vineland, New Jersey (5); and Milton, Wisconsin (6). All of these except the Lake George system and one slow-rate system were selected based on predetermined screening criteria by EPA (Table 1). Grants to evaluate the Lake George system and the San Angelo, Texas, system were already in progress when the decision was made to conduct a study of existing land application systems by contracts. Each of the other three sites listed above were selected from a number of candidate sites presented by the contractor. Final project selection was made after on-site field inspections by EPA personnel. These sites had been operating continuously for a minimum of 30 years, and one had been in service for as long as 50 years. Two of the sites were applying primary treated wastewater to infiltration beds, and two were applying secondary treated wastewater. This allowed a reasonable comparison of the treatment efficiency of two stages of pretreatment. None of the systems were chlorinating the treated effluent prior to application to infiltration basins.

Hollister, California

The project site is located in the San Juan Valley 35 kilometers (km) [22 miles (mi)] inland from Monterey Bay and about 144 km (90 mi) south of San Francisco. The City of Hollister and surrounding community collect sewage from a population of about 10,000. This collected sewage is piped to the treatment system which is located about 1.6 km (1 mi) west of the city, 150 meters (m) (500 ft) south of the San Benito river bed. A summary of general operating information for the systems at all four study sites is shown in Table 2.

Preapplication treatment at the Hollister wastewater treatment facility consists of primary clarification. Sludge from the clarifier is regularly drawn off and stored in a converted Imhoff tank, then periodically applied to sludge drying beds which are operated independent of the rapid infiltration basins.

Part of the influent wastewater flow is equalized in an excavated earthen reservoir before entering the head works and clarifier while the remainder is pumped directly to the clarifier. Wastewater is later pumped from the equalizer reservoir to the clarifier during the early morning hours when the flow into the equalizer is lowest.

TABLE 1. INITIAL SITE SCREENING CRITERIA

ITEM	CRITERIA
Period of operation	>10 years
Preapplication treatment	Remain unmodified for at least 10 years
Wastewater sources	Domestic and commercial[a]
Wastewater flowrate	>4.3 l/s
Wastewater application rate	>6 m/yr
Depth to groundwater	>3 m and <30 m
Control site	Comparable geohydrologic characteristics within 1.6 km of site
Availability of data	Historical wastewater and groundwater quality must be available for comparison purposes
Operation practices	1. Wastewater application to the spreading basins must be intermittent[b] 2. Sludge must never have been applied to the spreading basins[c] 3. Soil conditions in the basins should not have been altered drastically[d]

a. Industrial wastewater, in small amounts, resembling municipal wastewater is acceptable.

b. Systematic flooding and drying over several days with multiple independent basins.

c. Constituents are generally much more concentrated in sludge than in wastewater.

d. Surface disking or scarifying to restore infiltration is normal and acceptable.

l/s x 0.0228 = Mgal/d
m x 3.281 = ft
km x 0.621 = mi

TABLE 2. OPERATING FACTORS FOR EXISTING RAPID INFILTRATION SYSTEMS

FACTORS	HOLLISTER CALIFORNIA	LAKE GEORGE NEW YORK	VINELAND NEW JERSEY	MILTON WISCONSIN
Preapplication Treatment	Primary	Secondary	Primary	Secondary
Ground water level, m below surface	6.8-9.2[a]	1.3-7.0[a]	1.0-3.5[a]	2.0-3.0[a]
Hydraulic conductivity cm/hr	6.4-12.8	2.9[d]	15.8[d]	50[e]
Infiltration area, ha	8.8	2.2	36	0.2
Length of operation, yr	30	38	50	40
Total infiltration beds	20	21	32	1
Annual Hydraulic loading rate m/yr	15.4	44.7	11-21.4	110-224
Average daily flow, l/s	43.8	30.8	215	14.5
Industrial influence	Yes[b]	No	Yes[c]	No

a. Shallow water table created by infiltrating effluent.

b. Slaughterhouse = .9 l/s and paper recycle = 11.0 l/s.

c. Food processing wastewater is about 50% of Landis sewage authority flow.

d. Lowest value reported at the treatment site.

e. Estimated based on horizontal permeability.

The clarified effluent flows by gravity to the 20 individually controlled infiltration basins. The total basin area is 8.8 hectares (ha) [21.7 acres (ac)], and the individual basins range from 0.3 to 0.7 ha (0.6 to 1.8 ac). The basins are normally filled to depths of 30 cm (12 in); the duration of flooding is controlled by the basin area. Two basins are usually flooded simulatneously 8 to 10 hours each day if the daily flow exceeds the capacity of any one basin. The interval between wastewater applications ranged from 14 to 21 days.

The interval between wastewater applications ranged from 14 to 21 days during the study period. The length of time a basin was flooded and the interval between applications decreased in the cooler and wet winter months by 25 to 30 percent.

The effluent from the pretreatment facility was completely detained at the infiltration basin to which it was applied, leaving the site only by percolation and evaporation. The initial percolation rate of effluent was greater at the beginning of flooding than during the final stage because of the initial dryness of the soil and because clogging of the soil surface by effluent solids gradually reduces the infiltration rate. The average infiltration rate during the study period was 10 cm/day (4 in/day) when the basins were flooded to a depth of 30 cm (12 in).

The infiltration basins are located on alluvial deposits of the San Benito River characterized by the Soil Conservation Service (SCS) as Metz sandy loam. The permeability of this soil is reported to range from 6.4 to 12.8 cm/hr (2.5 to 5 in/hr) in the upper 3 m (10 ft) and 12.8 cm/hr to 25.4 cm/hr (5.0 to 10.0 in/hr) through the next 7.5 m (25 ft). The water table of the regional ground-water aquifer occurs at a depth of 20 m (65 ft) at the site. However, the saturated zone beneath the infiltration basins occurs at depths of 2 to 3 m (7 to 10 ft).

Lake George, New York

The Lake George sewage treatment plant was originally constructed in 1936, put into operation in 1939, and has operated continuously since that time. The plant is located approximately 1.6 kilometers (km) (1 mi) southwest of Lake George in the southeast corner of the New York State Adirondack Park. The treatment plant receives influent from two sources--Lake George Village and the town of Lake George. The plant was originally built to treat flows in the summer that were three times larger than winter flows. However, today the ratio is approximately two to one. Two Parshall flumes record the inflow from the village and town pumping stations.

Primary treatment consists of one circular Imhoff Tank and two mechanically cleaned circular settling digestion tanks. Secondary treatment is accomplished with two high rate rotating arm trickling filters in summer and one covered standard rate fixed nozzle sprinkling filter in winter. Two mechanically cleaned rectangular settling tanks and two circular settling tanks are used to complete secondary treatment. After secondary sedimentation, the treated sewage is discharged directly to the infiltration beds. A summary of general operating information for the system is presented in Table 2.

The treated sewage is carried by gravity to the 14 northern (lower) beds while the sewage is pumped to the newer seven southern (upper) beds. Normally, two beds are flooded simultaneously, one lower bed and one upper bed. Dosing is changed at approximately 8:00am and 4:00pm; thus, the entire day's flow is discharged to four beds. The applied effluent takes from 1/2 to 3 days to infiltrate below the soil surface, depending primarily on the size, age, and condition of the bed. The newer beds have higher infiltration rates than the older beds. Beds are maintained by removal of surface mats of entrapped suspended solids which inhibit infiltration. The cleaning schedule is not fixed, but based entirely on operator experience, availability of the bed for drying, and availability of plant personnel.

The infiltration beds in the Lake George system are located on delta deposits left by Pleistocene glaciation. These deposits are part of the Newland soil series which is composed of relatively fine uniform sands, low in organic matter. The average vertical permeability is 2.8 cm/hr (1.1 in/hr). Soil depth in the vicinity of the infiltration basins range from 2 to 9 m (7 to 30 ft) deep while the unsaturated zone ranges from 1.2 to 6 m (4 to 20 ft).

Vineland, New Jersey

The city of Vineland and the sewage treatment works are located in Cumberland County in southern New Jersey about 24 km (15 mi) from the northeastern shore of Delaware Bay which flows into the Atlantic Ocean. The Vineland sewage treatment works is actually composed of two separate treatment facilities operated by Borough of Vineland and the Landis Sewerage Authority. These systems are operated continuously and serve the city of Vineland and the surrounding area. The two facilities were considered as one test site even though historical length of operation and treatment of effluent are different.

The Vineland plant provides primary treatment of an average daily flow of 35 liters/second (l/s) [8×10^5 gallons per day (gpd)]. Primary treatment includes coarse screening of the influent and primary settling before being discharged directly to Basin I which contains a series of 19 infiltration beds.

The Landis Sewerage Authority's treatment plant received an average daily flow of 180 l/s [4.1 million gallons per day (mgd)] and provides primary treatment consisting of pre-aeration and primary settling before the treated effluent is discharged directly to two large infiltration basins (Basins II and III). A summary of the operation of the combined systems is presented in Table 2.

Basin I has been operated since 1928 by the Borough at Vineland. The Landis Sewerage Authority Basin II, containing a series of eight infiltration beds, has been in operation since 1948 and Basin III, containing a series of six infiltration beds, has been in operation since 1974. Basins I and II were operated by continuous flooding from the beginning of their use until April, 1973, when operation was changed to intermittent flooding. The

present operation is to flood the beds one at a time at each facility until the level of a bed reaches about 45 cm (18 in). When this level is reached, flow is diverted to the next bed. The length of time each bed is flooded varies, depending on volume of flow, bed size, and the soil percolation rate.

Normally after each loading, when the wastewater has percolated through the soil and the bed has dried, the entire bed is scarified to loosen the soil and restore the permeability of the top layer. Every six months each bed is plowed to a depth of 30 cm (12 in) to mix the soil and organic matter that has accumulated during percolation of wastewater.

The soils at the Vineland study site have been formed from materials of glacial origin and are characteristically deep, sandy, and well drained. Around the infiltration basins, soils belonging to the Eavesboro, Lakewood, and Lakehurst series are dominant. These series have relatively high permeabilities ranging from 5 to >20 cm/hr (2 to >6 in/hr), and low soil water holding capabilities. These soils are well drained, loose, and low in fertility and organic matter.

Milton, Wisconsin

The project site is located in Rock County, Wisconsin, which is in the south central part of the state about 64 km (40 mi) southeast of Madison. The Milton wastewater treatment plant gives secondary treatment followed by discharge to three lagoons constructed in series. The first two lagoons provide only detention time while the third lagoon is maintained as an infiltration basin. The lagoons are ccntinually flooded except as required to maintain infiltration, and the third lagoon is drained and cleaned by removal of the top few cm every two years. This material is replaced with fresh soil excavated from areas near the basin. The infiltration basin covers an area of approximately 1860 m^2 (20,000 ft^2) with an average annual inflow of 14.5 l/s ($3.3x10^5$ gpd) which results in a loading rate of 0.67 m (2.2 ft) per day.

The secondary treatment facilities evolved from the original facilities constructed in 1939 in an abandoned gravel pit. These original facilities consisted of activated sludge treatment and aerobic digestion using mechanical aerators. From 1939 to 1957 the effluent was passed through two rapid sand filters and then discharged thru dry wells in the surrounding gravel. The first lagoon in the series presently in use served for alternate disposal at the time. Sand filtration was discontinued in 1957, and the single lagoon was converted to two lagoons in series. In 1962, the original mechanical aerators were replaced with compressed air aeration, and the mechanical bar screen was replaced with a barminutor. Other additions included a blower building, primary tank bypass line, and larger capacity return sludge pumps. At about the same time, the third basin was constructed in series with the first two lagoons and used for infiltration. Operations have continued, as described, through the reported study period. General operating criteria are presented in Table 2.

The infiltration basin is located on Pleistocene glacial moraine sediments, 25 to 27 m (82-90 ft) deep at the site. These sediments generally consist of unsorted mixtures of clay, silt, sand, and gravel with stratified accumulations of sand and gravel. Beneath these glacial deposits are gray to yellow-gray dolomite, sandstone, and green shale of the Platteville-Galena unit of Ordovician age. The water table below the bottom of the infiltration basin normally is about 2 to 3 m (7 to 10 ft) while the control ground water saturated zones were a minimum of 15 to 18 m (50 to 60 ft) below ground level.

SECTION 5

COMPARISON OF APPLIED AND RENOVATED WASTEWATER

The four rapid infiltration systems reviewed in this summary have been in operation from 30 to 50 years, apparently without significant adverse environmental effects. Comparison of the summarized data from each of these sites along with some general interpretations of these data should give the designers and planners a better insight to the treatment capability of rapid infiltration systems where either primary or secondary treated effluent is applied to the infiltration basins. The ability of these high rate systems to remove most pollutants to near acceptable drinking water standards for both levels of pretreatment lends credibility to the use of such systems as a viable alternative for municipal wastewater treatment.

The four existing systems were historically constructed and operated based on hydraulic acceptability of the soil at specific sites, and then the systems were expanded as the volume of treated sewage required for infiltration increased. Little correlation can be made between those four sites with respect to their comparative treatment efficiency and systems operation due to these progressive changes. Until recent years, these four systems were primarily operated with disposal of wastewater in mind with little regard to treatment. Therefore, loading frequency and system management were quite variable between the systems. As noted in Section 4, systems operation varied from continuous flooding of primary treated wastewater at Vineland, New Jersey, and secondary treated wastewater at Milton, Wisconsin, to a very well scheduled cycle of flooding and drying of secondary treated wastewater at Lake George, New York, and primary treated wastewater at Hollister, California. All four of these systems were operated year-round, three of them during sub-freezing winter weather. Enough heat is normally retained in the treated wastewater (e.g. , the Lake George system) with the pretreatment utilized to thaw the soil and ice remaining from the previous flooding cycle. However, it was reported that during winter the hydraulic conductivity of infiltration basins is reduced by as much as 25 to 30 percent even at Hollister, California, which has the mildest climate of these four sites. This change corresponds to expected changes due to temperature effects on the viscosity of water. Data of the difference in hydraulic conductivity between winter and summer operations at the other three sites were not reported.

It is common knowledge to researchers conducting studies of land application systems that nitrification and denitrification during cold winter months is normally reduced considerably from summer operations. This is

primarily because the organism activity is reduced by cold temperatures. Therefore, during winter operation a greater mass of nitrogen is infiltrated into the ground water. The data presented in this summary are averages for the entire study period, except for Lake George which reported the range of each parameter for seasonal extremes during the study period. The data, therefore, do not show seasonal changes as discussed above for all four sites, and reflections of seasonal influences cannot be verified.

The most practical way to evaluate the treatment efficiency of these systems is to compare the chemical characteristics of the wastewater applied to the infiltration beds to water collected from the top 2 to 20 m (6 to 60 ft) of ground water below or near these beds as shown in Table 3. These data should then be compared to data collected from the native ground water at remote control wells located up-gradient and out of the influence of the infiltration beds (Table 4). Comparison of the data from these three points of interest accurately reveals which parameters are receiving treatment and which are reaching the ground water essentially untreated. In a review of the reports of individual systems, it can be seen that down-gradient wells indicate certain parameters remain elevated above those of the up-gradient control wells for several hundred feet before differences in concentration are insignificant.

In evaluating the data, the reviewers must keep in mind there are a number of conditions that influence the treatment efficiency at all of the sites such as effluent pretreatment, rate of infiltration, soil chemical characteristics, and depth of unsaturated soil. The most obvious effect of pretreatment is shown in Table 3 comparing the chemical oxygen demand (COD), biochemical oxygen demand (BOD), total organic carbon (TOC), total nitrogen (total-N), organic nitrogen (organic-N), and nitrate nitrogen (nitrate-N) for the four systems. The Hollister and Vineland sites receive only primary treatment before the effluent is applied to the infiltration beds while the other two sites receive secondary treatment. There is a significant contrast in the concentration of constituents applied to the beds. Comparison of these parameters in Table 3 all indicate water recovered from shallow ground water wells beneath or near the infiltration basins is greatly improved from that applied to the basins from both types of pretreated effluent. However, the percent removal is significantly higher in systems applying primary treated wastewater than systems applying secondary wastewater.

When the quality of the shallow ground water wells is compared to that of the control wells (Table 4), it is obvious that these constituents are not completely removed before entering the upper levels of the ground water. Even though complete treatment is seldom accomplished, it can be seen by review of the entire list of parameters that drinking water quality standards are generally met for both levels of pretreated effluent with the exception of coliforms.

The removal of nitrogen from the wastewater in land application systems is a multi-step operation. Oxidation of the organic and ammonia nitrogen is the first step in the process. Comparsion of the data in Tables 3 and 4

TABLE 3. COMPARISON OF APPLIED WASTEWATER VS. SHALLOW GROUND WATER

	HOLLISTER, CA		LAKE GEORGE, NY		VINELAND, NJ		MILTON, WI	
	Effluent[a]	Shallow Ground Water[a]	Effluent[b]	Shallow Ground Water[b]	Effluent[c]	Shallow Ground Water[c]	Effluent[a]	Shallow Ground Water[a]
COD	706	50			372-287	75-99	84	36
BOD	220	13	23-40	0.8-3.0	154-149	6.5-12	28.0	5.2
TOC	248	11			108-82		36	-
Total-N	40.2	1.7	12.2-33.5	.24-7.5	40.4-38.8	23.7-28.6	30.8	21.8
NH_3-N	25.3	<0.1	1.8-15.8	.024-0.12	17.2-19.0	11.6-17.5	19	20.3
Kjel-N			11.7-27.6	.10-.49				
Organic-N	14.5	1.0			23.1-18.5		6.5	-
Sol. Org.-N					16.8-21.1	10.6-11.0	1.3	0.7
Nitrate-N	0.43	0.6	0.5-5.9	.14-7.0	0.1-1.3	1.49-0.06	5.3	0.8
Total-P	12.4	9.6	2.0-4.5	.32-1.08	9.3-9.0		8.2	-
Ortho-P	10.5	8.7	0.4-2.0	.004-.43	4.8-4.8	1.54-3.8	7.0	5.8
pH	7.3	7.7	6.6-7.6	6.5-7.4	6.5-6.8	6.6-6.9	7.7	7.5
TDS	1208	1275	175-240	84-175	214-304	266-381	728	703
SS	274	10			43-41		29	-
VSS					27-29		21	-
Conductivity (μmhos/cm)	1790	1828			548-602	468-688	1431	1379
Alk	446	522	80-124	43.5-94	115-130	119-187	410	427
Total-Coli (#/100ml)	27.6×10^6	0.231×10^6	$.3-.7 \times 10^6$	-	TNTC	TNTC	TNTC	TNTC
Fecal-Coli (#/100ml)	12.4×10^6	0.186×10^6	$.225-.525 \times 10^6$	<1	TNTC	TNTC	TNTC	476
Na	262	261	10-17	5.7-7.4	60-75	38-102	147	13.5
K	12.9	14.9	4-6	0.87-1.2	14.5-10.4	10.6-13.3	14	13.5
Ca	54	107	14-22	10.2-14.6	12-11	14-9	70	71
Mg	64	74	5-6.1	3.7-7.2	4-4	5-4	37	40
B	1.4	1.2			.311-.534	.332-.593	.681	.628

(continued)

TABLE 3. (continued)

	HOLLISTER, CA		LAKE GEORGE, NY		VINELAND, NJ		MILTON, WI	
	Effluent[a]	Shallow Ground Water[a]	Effluent[b]	Shallow Ground Water[b]	Effluent[c]	Shallow Ground Water[c]	Effluent[a]	Shallow Ground Water[a]
Cl	284	292	32-101	29.5-64	57-77	27-79	200	179
Sulfide					<.1-<.1	5.7-16.5	0.1	0.5
Sulfate	213	247			14-24	22-19	51	32
As	<0.01	<0.01			<.005-<.005	.01-.008	.011	.033
Fe	0.39	0.51	0.2-.85	.43-8.25	1.5-1.2	12.6-9.0	.50	2.0
Hg	<0.001	<0.001			<.001-<.001	.001-.001	.001	.001
Mn	0.070	0.62			<.06-<.07	.13-.09	.10	.29
Ni	0.051	0.16			<.1-<.1	.1-.1	.10	.10
Pb	0.054	0.08			<.1-<.1	.1-.1	.2	.2
Sc	<0.001	<0.001			<.01-<.01	.01-.01	.012	.005
Zn	0.048	0.090			.127-.121	.024-.043	.078	.254
Ba	<0.13	<0.15						
Cd	<0.004	0.050			<.02-<.02	.02-.02	.02	.02
Co	<0.008	0.012			<.05-<.05	.05-.05	.05	.05
Cr	<0.014	<0.017			<.02-<.02	.02-.02	.05	.05
Cu	.034	0.036			.058-.077	.023-.02	.10	.10
Al					.6-.6	.5-.5	0.3	1.0
Endrin (ng/l)					<.03-<.03	.03-.03	.03	.03
Lindane (ng/l)					2830-1227	453-1173	41	157.6
Methoxychlor (ng/l)					<.01-<.01	.01-.01	0.01	0.01
Toxaphene (ng/l)					<.1-<.1	.1-.1	0.1	0.1
2,4-D (ng/l)					9.5-10.5	16.4-13.0	53.8	92.4
2,4,5-TP silvex (ng/l)					72-72	26.8-120	16.2	41.2

l = all values in ng/l unless otherwise noted.
a = mean values, Effluent is input to land treatment system.
b= mean seasonal range
c= Vineland and Landis sites mean values, respectively.

TABLE 4. COMPARISON OF SHALLOW GROUND WATER[1] TO CONTROL WELLS[2]

	HOLLISTER, CA		LAKE GEORGE, NY		VINELAND, NJ		MILTON, WI	
	Control Well[a]	Shallow Ground Water[a]	Control Well[b]	Shallow Ground Water[b]	Control Well[a]	Shallow Ground Water[c]	Control Well[a]	Shallow Ground Water[a]
COD	16	50	-	-	7	75-99	18.0	36
BOD	3	13	1.2	0.8-3.0	1.1	6.5-12	3.9	5.2
TOC	<1	11	-	-	-	-	-	-
Total-N	2.1	1.7	.55-1.7	0.24-7.5	3.7	23.7-28.6	-	-
NH_3-N	<0.1	<0.1	.035-.087	.024-0.12	0.1	11.6-17.5	0.2	20.3
Kjel-N	-	-	.53-1.66	.10-.49	-	-	-	-
Organic-N	< 0.5	1.0	-	-	-	-	-	-
Sol. Org.-N	-	-	-	-	1.8	10.6-11.0	0.5	.7
Nitrate-N	1.9	0.6	.019-.053	.14-7.0	1.0	1.49-0.06	6.7	.8
Total-P	0.1	9.6	.078-.347	.32-1.08	-	-	-	-
Ortho-P	0.04	8.7	<.001-.001	.004-.43	0.03	1.54-3.8	0.05	5.8
pH	7.8	7.7	6.6-7.8	6.5-7.4	5.2	6.6-6.9	7.7	7.5
TDS	820	1275	115-145	84-175	40	266-381	405	703
SS	-	10	-	-	-	-	-	-
VSS	-	-	-	-	-	-	-	-
Conductivity (µmhos/cm)	1148	1828	-	-	64	468-688	677	1379
Alk	425	522	61-93	43.5-94	7	119-187	282	427
Total-Coli (#/100ml)	<2	0.231×10^6	<1	-	51	TNTC	17	TNTC
Fecal-Coli (#/100ml)	0	$.186 \times 10^6$	<1	<1	0	TNTC	0	476

(continued)

TABLE 4. (continued)

	HOLLISTER, CA		LAKE GEORGE, NY		VINELAND, NJ		MILTON, WI	
	Control Well[a]	Shallow Ground Water[a]	Control Well[b]	Shallow Ground Water[b]	Control Well[a]	Shallow Ground Water[c]	Control Well[a]	Shallow Ground Water[a]
Na	112	261	-	5.7-7.4	2	38-102	9	135
K	4.1	14.9	-	0.87-1.2	1.2	10.6-13.3	2.1	13.5
Ca	47	107	-	10.2-14.6	3	14-9	84	71
Mg	89	74	-	3.7-7.2	1	5-4	37	40
B	0.7	1.2	-	-	.113	.332-.593	.248	.628
Cl	76	292	32-64.4	29.5-64	4	27-79	23	179
Sulfide	-	-	-	-	0.1	5.7-16.5	0.1	.5
Sulfate	184	247	-	-	9	22-19	43.0	32
As	<0.01	<0.01	-	-	.006	.01-.008	.005	.033
Fe	0.03	<0.51	2.24	.43-8.25	1.6	12.6-9.0	.5	2.0
Hg	<0.001	<0.001	-	-	.001	.001	.001	.001
Mn	0.01	.62	-	-	.11	.13-.09	0.10	.29
Ni	0.039	.16	-	-	.10	<.1-.1	.10	.10
Pb	0.012	.08	-	-	.10	.1-.1	.2	.2
Sc	<0.001	<0.001	-	-	.01	.01-.01	.05	.005
Zn	0.05	0.090	-	-	.043	.024-.043	.628	.254
Ba	<0.07	<0.15	-	-	-	-	-	-
Cd	<0.003	0.050	-	-	.02	.02-.02	.02	.02
Co	<0.006	0.013	-	-	.05	.05-.05	.05	.05
Cr	0.032	<0.017	-	-	.02	.02-.02	.05	.05
Cu	0.025	0.036	-	-	.02	.023-.02	.10	.10
Al	-	-	-	-	1.8	.5-.5	1.0	1.0

(continued)

TABLE 4. (continued)

	HOLLISTER, CA		LAKE GEORGE, NY		VINELAND, NJ		MILTON, WI	
	Control Well[a]	Shallow Ground Water[a]	Control Well[b]	Shallow Ground Water[b]	Control Well[a]	Shallow Ground Water[c]	Control Well[a]	Shallow Ground Water[a]
Endrin (ng/l)	-	-	-	-	.03	.03-.03	.03	.03
Lindane (ng/l)	-	-	-	-	21.3	453-1173	7.4	157.6
Methoxychlor (ng/l)	-	-	-	-	0.01	.01-.01	.01	.01
Toxaphene (ng/l)	-	-	-	-	0.1	.1-.1	.1	.1
2,4-D (ng/l)	-	-	-	-	10.4	16.4-13.0	31.0	92.4
2,4,5-TP silvex (ng/l)	-	-	-	-	185	26.8-120	76.8	41.2

1 - averages of data from well sampled in the top 2 to 20 m (6 to 60 ft) of ground water. All values in mg/l unless otherwise noted.
2 - averages of data from deep aquifer wells sampled up gradient out of the influence of infiltration beds. All values in mg/l unless otherwise noted.
a - mean values
b - mean seasonal range
c - Vineland and Landis sites mean values, respectively.

indicate Hollister and Lake George were effective in this oxidation process but Vineland and Milton did not effectively oxidize the nitrogen. The probable causes for this ineffective treatment is insufficient oxygen in the soil profile. A high water table where oxygen could not enter the soil profile during drying and long flooding cycles where the available oxygen is consumed before a new supply is made available could cause this response. Both of these conditions existed at Vineland and Milton. This problem can be alleviated by changing the cyclic operation allowing additional drying time, underdraining the soil profile to allow for additional storage of oxygen between applications, and/or reducing the application rate.

The two major fractions of phosphorus involved in the land treatment process are soluble and insoluble phosphorus. The insoluble phosphorus is physically filtered by the soil and degraded biologically near the surface forming soluble phosphorus compounds. These compounds combine with soluble phosphorus initially in the wastewater to move with the water into the soil profile. Once in the soil profile, two primary factors control the fate of the phosphorus. First, each soil has an ultimate capacity to retain phosphorus. When the capacity is reached, phosphorus will move through this portion of the profile unattenuated. The second, and probably more important factor, is the time available for the soil to react with the phosphorus. Table 3 indicates the total capacity for phosphorus removal by the profile has not been reached at any of the sites evaluated. Since the concentration in the shallow ground water is consistently lower than the applied effluent, the removal efficiency is quite variable due to differences in detention in the profile. This aspect is discussed in more detail in Section 7. An example of the soil's capability for phosphorus removal was demonstrated at Lake George, which had the lowest infiltration rate of the four systems evaluated and 38 years of operation. Lake George, which had an unsaturated soil depth of 1.3 to 7.0 m (4 to 20 ft), showed approximately 80 percent removal of both total and ortho-phosphate, indicating soil depth coupled with detention time through the profile are necessary for good phosphorus sorption efficiency.

The total dissolved solids (TDS) were quite variable between the sites. Evaporation from the surface can be a factor in increasing the concentration of ions remaining in the waste stream and will increase the TDS. However, in the operation of rapid infiltration systems, concentration changes due to evaporation should be minimal due to the high hydraulic loading compared to evaporation. Evaporation from basins in high rate systems seldom exceed 1 cm/day, even in hot dry climates. Dissolution of minerals is a potential source for an increase in the TDS, particularly when the carbon dioxide (CO_2) partial pressure is increased due to degradation of organic material. This phenomenon was observed in Tables 3 and 4 at Hollister where calcium (Ca) increased and was further verified by a decrease in calcium carbonate ($CaCO_3$) in the profile as dicussed in Section 6. Increases due to similar mechanisms were observed for iron (Fe) as discussed later. On the other hand, data from Lake George indicated a significant retention of cations (Na, Ca, K, and Mg). The reduction in cations corresponds to a reduction in TDS and conductivity at the site (Tables 3 and 4).

Fecal coliform counts were generally reduced by two orders of magnitude except in the Vineland system which applied primary effluent containing coliforms that were too numerous to count (TNTC) through soils only 1.0 to 3.5 m (3 to 10 ft) thick (Table 3). Both total and fecal coliforms at this site were also TNTC in the shallow ground water beneath the infiltration beds indicating very poor treatment (Table 4). The Milton system with a similar shallow water table also experienced high total coliform counts in the ground water. However, total coliform counts in soil is not necessarily indicative of pollution but more of biological activity.

Even though 6.8 to 9.2 m (22 to 30 ft) of unsaturated soil profile exist at Hollister, significantly higher total and fecal coliform counts were found in the shallow ground water as compared to the control wells (Table 4). These data indicate that renovated water should be allowed to move laterally a sufficient distance through the aquifer to obtain coliform-free renovated water.

Thirteen metals were measured on the applied wastewater and ground water beneath the basins at all sites and control wells with the exception of Lake George where only Fe was measured. All these metals were in very low concentrations with the exception of Fe, manganese (Mn), and zinc (Zn). The Fe data indicated significant leaching from the soil profile at all four sites as shown in Tables 3 and 4. The behavior of Mn at the three sites where it was measured was very similar to that of Fe. Data indicate Zn was leached from the soil at Hollister and Milton, but was accumulated in the upper soil profile at Vineland. Soil leaching of arsenic (As) was evident at Milton when the data of the shallow ground water wells are compared to the control wells. The mechanism for leaching of these specific metals is reported to be the reduction of oxides and hydroxides of the complex in reducing environment to more soluble forms and/or chelation by the organic constituents remaining in the water (2).

Six pesticides were measured in the applied effluent and ground water at the Vineland and Milton sites. These include endrin, lindane, methoxychlor, toxaphene, 2,4-D, and 2,4,5-TP silvex. Endrin, methoxychlor, and toxaphene were not found above the respective detectable limits of <0.03, <0.01, and <0.1 nanogrmas/liter (ng/l) at either site.

The herbicides 2,4-D and 2,4,5-TP silvex were found at concentrations in the same range as encountered in the control ground water. The National Interim Primary Drinking Water Regulations allow maximum concentrations of 100,000 ng/l and 10,000 ng/l for 2,4-D and 2,4,5-TP silvex, respectively, in domestic water supplies (2). The measured values of these herbicides were insignificant compared to the standard and should be of no concern.

Lindane concentrations in the ground water beneath the infiltration beds ranged from maximums of 6480 ng/l to 1360 ng/l for the Vineland and Milton sites, respectively. The average concentration of lindane in the ground water at these sites was 453 to 1173 ng/l beneath the Vineland basins and 157.6 ng/l beneath the Milton basin. The National Interim Primary Drinking Water

Regulations standard maximum contaminated level for lindane is 4000 ng/l (2). The only system in which there appeared to be any treatment of the pesticides or herbicides was at Vineland Basin I in which about 85 percent of the lindane was removed. This system had been in operation for 50 years but had no record of how long wastewater, high in lindane, was applied. Basin II, which had been in service for about 30 years, had a reduction of only about 5 percent of the lindane as a result of the treatment process (Table 3). Lindane reaching the ground water remained significantly higher than the control wells.

SECTION 6

OBSERVED CHANGES IN SOIL CHEMICAL PROPERTIES

Describing the response of the pH at a rapid infiltration system is extremely complex. There are several factors which tend to increase the acidity and several factors which tend to decrease the acidity. In the process of organic matter decomposition, both organic and inorganic acids are formed. The simplest and, perhaps the most widely found, is carbonic acid (H_2CO_3) which results from the reaction of CO_2 and water. Soils with high water contents and ample supply or organic carbon tend to have elevated CO_2 partial pressure and greater potential to form acids. Since organic matter decomposition is a biological process, temperature of the system indirectly affects the pH. The organic carbon is also attacked by fungi which have among their metabolic end products strong complex organic acids. On the other hand, supplying dissolved solids in the wastewater changes the ionic balance supplying metals to the exchange sites replacing hydrogen. This has a tendency to increase the pH of the soil.

The actual pH of the operating systems was not measured. Slight changes in the CO_2 partial pressure will result in significant changes in the measured pH. The reported results indicate changes in base saturation of the soil profile rather than hydrogen ion activity of the operating system. Figure 1 shows the changes that were observed at the Hollister, California, and Vineland, New Jersey, sites. The results indicate the systems are reequilibrating near neutrality throughout the 500 cm (17 ft) depth of monitoring.

Metals are known to be adsorbed by the soil matrix and to precipitate forming relatively insoluble compounds. In general, the more acidic the system, the more soluble the compounds. For this reason, toxicity to metals tends to occur more frequently in acidic conditions, and the mobility of the metal is greater under acidic conditions.

Two metals were monitored which showed significant accumulations in the soil profile, zinc (Zn) in Figure 2 and copper (Cu) in Figure 3. In both cases there is significantly more accumulation at the surface than there is with increasing depth. Also, the more acidic sites show more accumulation with depth. These observations are consistent with theoretical expectations, and one would not anticipate encountering difficulties with metal toxicity unless acidic conditions develop or occur during system operation.

In some respects, phosphates behave similarly to metals in that sorption and precipitation reactions are involved. The major difference is that maximum solubility of phosphate compounds occur near neutral conditions. One

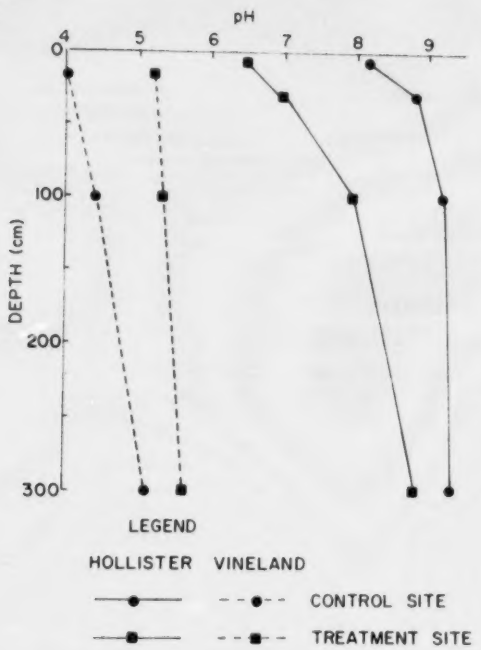

Figure 1. pH comparisons in the soil profile.

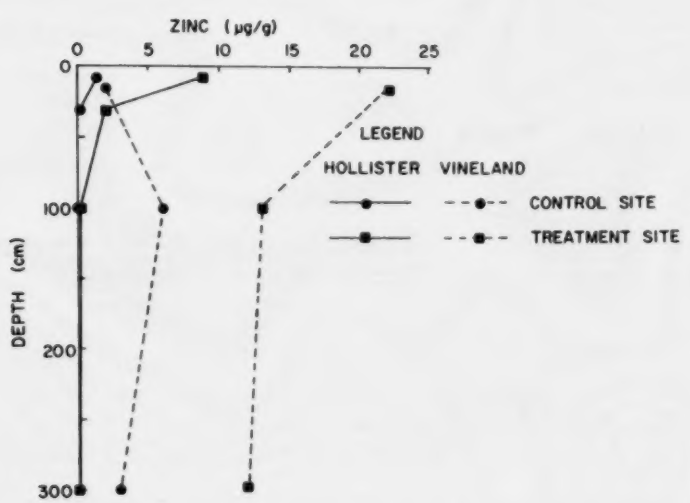

Figure 2. Zinc concentrations in the soil profile.

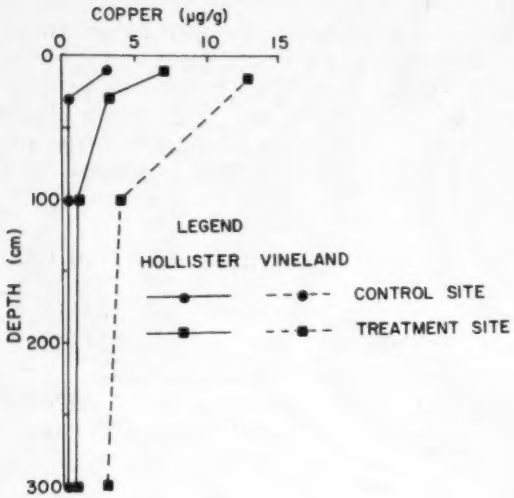

Figure 3. Copper concentrations in the soil profile. Copper concentrations at the Vineland control site were below detectable limits (<2µg/g).

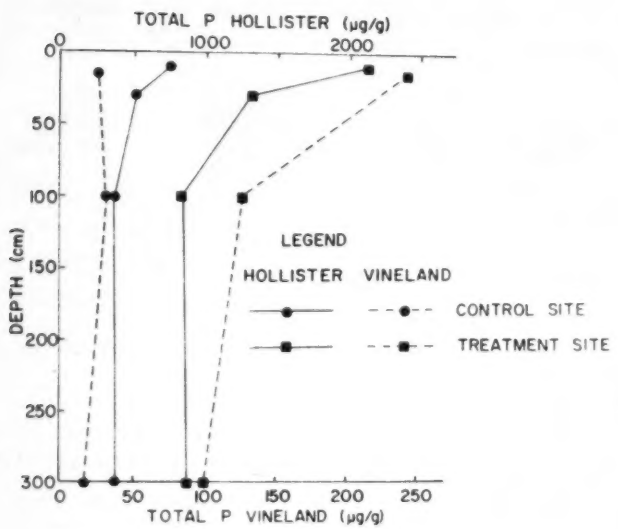

Figure 4. Total phosphate comparisons in the soil profile.

would, therefore, expect, under similar loading conditions, greater penetration in neutral soils. Figure 4 shows the total phosphorus sorbed or precipitated in the solid phase as a function of depth for the two sites monitored. Each show significantly more accumulation at the surface than at greater depths. Both also show significant accumulation throughout the depth monitored. Since phosphorus is not a toxic compound, significant accumulations should not be considered detrimental to future system operation. Since the soil does have a finite capacity to react with phosphorus, the depth of penetration can be expected to increase with time and the amount entering the receiving body to increase once the capacity of the surface soil is reached.

In those soils where $CaCO_3$ is present, the increased CO_2 partial pressure, caused by microbiological activity and high water contents, tends to dissolve the $CaCO_3$ minerals. Major reductions throughout the profile were observed in $CaCO_3$ at the Hollister site as shown in Figure 5. Hollister was the only site where this mineral was reported. Losing the $CaCO_3$ would tend to increase the TDS and hardness in the reclaimed water and reduce the capacity of the profile to react with phosphorus where dicalcium phosphate dihydrate ($CaHPO_4 \cdot 2H_2O$) is a dominant reaction product under calcareous conditions.

Organic nitrogen build-up was all in the surface layers of the soil as shown in Figure 6. The only significant differences were in the surface most sampling of the soil profile. This observation is most likely due to the physical filtering of the suspended organic material which is high in organic nitrogen. Once a system is abandoned one would anticipate this observed increase in organic nitrogen to revert to background conditions due to biological degradation of the organic materials.

SUMMARY

Long-term operation of rapid infiltration wastewater treatment systems have significant impact on chemical properties of the soils where the wastewater is applied. But the changes observed are not necessarily detrimental to man's existence or to his environment. The following general observations can be made:

1) Operation of the treatment system tends to drive the pH of the soil toward neutrality.

2) Elements which form relatively insoluble compounds tend to accumulate near the soil surface.

3) Organic materials which can be filtered from the solution accumulate until they are degraded as evidenced by accumulation of organic nitrogen.

4) Relatively soluble minerals such as $CaCO_3$ may be leached from the profile.

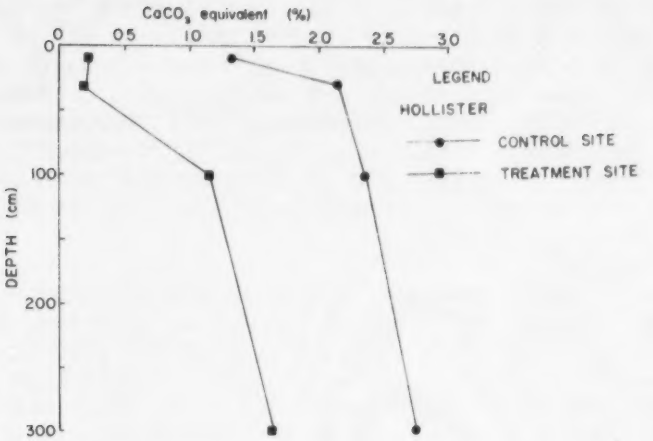

Figure 5. Calcium carbonate reduction in the soil profile.

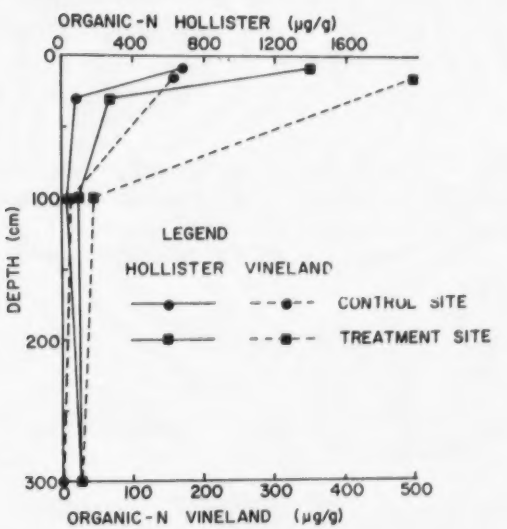

Figure 6. Organic nitrogen reduction comparisons in the soil profile.

SECTION 7

RATIONALE FOR RAPID INFILTRATION DESIGN

The design of a rapid infiltration wastewater system is based on determining the most limiting criteria. The design can be based on either hydraulic loading limitations or based on water quality leaving the treatment system. A preliminary rationale is presented for selected parameters. The rationale is then tested at the four existing systems and design results compared with actual practice.

HYDRAULIC LOADING

The Process Design Manual for Land Treatment of Municipal Wastewater (1) bases rapid infiltration (RI) design on hydraulic acceptability. The four parameters required to evaluate the suitability of the wastewater and determine the maximum hydraulic loading for hydraulic acceptability are:

1) the SAR of the applied wastewater

2) the saturated vertical hyraulic conductivity of the most hydraulically restrictive soil layer

3) the BOD of the applied wastewater and

4) the total suspended solids (TSS) of the applied wastewater

The sodium adsorption ratio is defined by the equation:

$$SAR = \frac{Na^{+}}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} \qquad (1)$$

where the concentrations are expressed in milliequivalents per liter. This parameter is important because of its tendency to promote conditions favorable to flocculation or deflocculation of the soil particles and thereby affects the soil's capability to accept and transmit water. This property is more important in fine textured soils than those normally associated with rapid infiltration systems. In the design of rapid infiltration systems, the SAR will be a severely limiting factor if the value is greater than 9 (1). A value of 15 should be considered as a practical upper limit for the SAR.

The most difficult parameter to accurately evaluate is the hydraulic conductivity of the soil profile. The Process Design Manual for Land Treatment of Municipal Wastewater (1) specifies design based on the hydraulic conductivity (sometimes called permeability or transmissivity) of the most restrictive layer. A more rational approach would be to consider the geometric mean of individual measurements of the hydraulic conductivity from the entire profile. This would permit considering the effect of a small perched water table increasing the hydraulic gradient across the restricting layer. Infiltrometers are often used to estimate the hydraulic conductivity under the assumption that when the infiltration rate reaches a constant value, gravity will be the dominate force causing flow, thereby creating a unit gradient. This would make the infiltration rate approximately equal to the saturated hydraulic conductivity. Using the double ring cylindrical infiltrometer [as described in Section C.3.1.4.3 (1)] or the U.S. Public Health Service percolation test for septic tanks (7) can give highly erroneous results. The U.S. Public Health Service percolation test does not consider the correct geometry for large systems while the infiltrometer may lead to an inaccurate evaluation due to spatial variability. Field studies have shown as much as an order of magnitude variation within a single mapping unit on a single field using the infiltrometer. The best currently available technique to evaluate the hydraulic conductivity is the sprinkler infiltromter [described in C.3.1.4.2 (1)] which averages a relatively large area. In this technique, water is not ponded on the surface which eliminates the flow through worm holes and cracks. These channels should not be considered when evaluating the hydraulic conductivity. Where a sprinkler infiltrometer is not available, the basin flooding method [described in C.3.1.4.1 (1)] should be used since it will give the average for a reasonably large area. A maximum hydraulic loading of 10 percent of the saturated hydraulic conductivity of the most restrictive layer has been suggested [Figure 3-5 (1)]. This limit can also be modified by the BOD and SS of the applied wastewater. A maximum of 2.24 milligrams per square centimeter/day ($mg/cm^2/day$) [200 lb/ac/day] for BOD and SS is rarely exceeded [pg. 5-8 (1)]. With loadings in this range one would anticipate after passing thru 4.5 m (15 ft) of soil, the BOD and SS would be less than or equal to 5 mg/l [pg. 2-4 (1)].

This analysis procedure assures the designer that the soil will accept the hydraulic load. However, no assurance is made from the analysis that a ground water mound will not develop, thereby impairing the operation of the system. Maintaining a minimum of 1.5 m (5 ft) of unsaturated soil profile is recommended (1). Although higher water tables are permitted during flooding, the designer must further evaluate the system's geohydrology and be assured of adequate drainage, either by natural or man-made systems.

WATER QUALITY LIMITATIONS

The Process Design Manual (1) does not give the consulting engineer a procedure to estimate the percolate water quality. The manual only indicates 15 to 25 percent of the nitrogen will be lost. There are two ways to consider nitrogen transformation. One is the form of nitrogen (nitrate, ammonia, or organic-N) reaching the ground water or being discharged from the soil treatment system. The alternate approach is to consider the total nitrogen discharged to either the ground water or another receiving body of water.

Loading to Obtain Nitrification

When the receiving body of water is ammonia limited, the maximum loading rate is limited by the nitrification rate. Nitrification is a process of enzymatic oxidation brought about by a specialized group of bacteria. Nitrifying bacteria are present in almost all soils and can remain active over a wide range of moisture and temperature conditions. These autotrophic bacteria obtain their energy by the oxidation of NH_4^+ to NO_3^-. The bacteria are obligate aerobes but are capable of functioning at oxygen concentrations significantly lower than atmospheric levels. The complete reaction converting NH_4^+ to NO_3^- takes place in two steps. Nitrosomonas and Nitrosoccoccus species are concerned with conversion of ammonium to nitrite following the reaction:

$$2NH_4^+ + 3O_2 \rightarrow 2NO_2^- + 2H_2O + 4H^+ + \text{Energy}$$

The oxidation of nitrite to nitrate is controlled by Nitrobacteria species following the reaction:

$$2NO_2^- + O_2 \rightarrow 2NO_3^- + \text{Energy}$$

This latter reaction generally proceeds rapidly. Significant levels of NO_2^- are, therefore, rarely found in soil solution.

Nitrifying bacteria are extremely sensitive to their environment. Where nitrification is the objective of the treatment process, activities promoting nitrification should be encouraged. Since nitrification is a process of oxidation, activities increasing soil oxygen should, up to a limiting reaction rate, encourage bacterial activity. Drying or resting and cultivation are recognized methods of promoting soil nitrification. The rate of nitrification is directly proportional to the temperature. Increasing the temperature increases the rate of nitrification throughout the temperature range found at rapid infiltration treatment systems. Figure 7 shows the limiting rate of nitrification, as a function of temperature assuming a neutral pH.

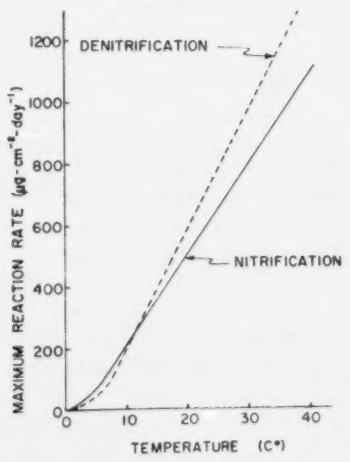

Figure 7. Nitrification and denitrification versus temperature.

An approximate relationship to estimate maximum hydraulic loading and maintain the desired level of nitrification can be described by the equation:

$$LR_{NO_3} = \frac{30\ (T-3)}{X} \qquad (2)$$

where LR_{NO_3} = maximum daily loading expressed in cm/day to obtain the desired level of nitrification

T = temperature of the applied wastewater in C^o. This may be approximated as mean air when the system is operated continuously and lagoon pretreatment is not employed.

X = total kjeldahl nitrogen applied minus the design TKN reaching the ground water or discharge point (measured in mg/l).

Equation 2 is believed to be valid as long as the TKN reaching the ground water is greater than or equal to 1 mg/l. Maintaining a concentration of less than 1 mg/l is extremely difficult and should not be considered practical through rapid infiltration technology. The rate of nitrification is also affected by pH. Below a pH of 4.5, nitrification ceases: However, since most RI systems operate near neutral conditions, this rarely is a problem.

Loading to Obtain Denitrification

Nitrogen removal is more complex than nitrification alone, since both nitrification and denitrification processes are involved. The denitrification process is a biochemical process that reduces nitrate and nitrite to ammonia, nitrous oxide (N_2O) and molecular nitrogen (N_2). Denitrifying bacteria use the oxygen to nitrate and nitrite in their oxidation of organic matter. Several bacteria are capable of accomplishing this process. From a land treatment point of view, the most important transformation involves the conversion of nitrate to the gaseous forms N_2O and N_2. The process can be described by the reaction:

$$C_6H_{12}O_6 + 4NO_3^- \rightarrow 2N_2 \uparrow + 6H_2O + 6CO_2$$

Denitrifying bacteria, like nitrifying bacteria, are sensitive to their environment. Denitrification is inhibited by oxygen. Processes reducing soil oxygen, up to a limiting rate, encourage denitrification. The more reducing the soil environment is the greater the denitrification rate. In wastewater studies, denitrification rates exceeding 20 kg/ha/day (110 lb/ac/day) have been observed.

Denitrification requires organic carbon. The potential carbon limitation at a land treatment site using municipal wastewater has been described by the equation:

$$\Delta N = \frac{TOC-K}{2} \qquad (3)$$

where ΔN = change in total nitrogen (mg/l)
TOC = total organic carbon in the applied wastewater (mg/l)
K = TOC remaining after wastewater treatment which is typically 5 for municipal wastewater after passing through 1.5 m (5 ft) of soil during summer operation. This value increases during cold weather (8).

The coefficient two (2) in the equation was based on experimental data where two grams of wastewater carbon were required to denitrify one gram of wastewater nitrogen (9). For nitrate nitrogen, 1 mg/l should be considered the lowest concentration consistantly obtainable through RI.

In addition to requiring sufficient carbon for the reaction, the denitrifying process may be rate limiting. Thus, even if sufficient carbon is available, adequate treatment may not be achieved if sufficient time is not allowed for the reactions to take place. Figure 7 graphically shows the effect of temperature on the reaction. In terms of maximum loading rate to achieve the desired denitrification, an approximate relationship can be given as:

$$LR_{DN} = \frac{40\ (T-5)}{Y} \qquad (4)$$

where LR_{DN} = maximum daily loading expressed in cm/day to obtain the desired denitrification
Y = nitrate applied plus the nitrate converted from Equation 2 minus the nitrate entering ground water (all in mg/l).

Ammonia Volatilization Considerations

The ammonia reported in water quality standards is primarily in the form of an ammonium ion. As the pH rises, there is a higher percentage of ammonia. Management which promotes high pH followed by conditions conducive to volatilization of the ammonia to the atmosphere such as sprinkler operation help to lower the ammonia applied at the land treatment site. Volatilization plus denitrification is estimated to be responsible for the removal of at least 15 to 25 percent of the applied nitrogen (1). A portion of this loss is due to the volatilization process. But most researchers believe this to be of minor importance when considering land application of municipal wastewaters following conventional pretreatment.

Transport of Phosphate

Predicting changes in phosphorus concentration as wastewater passes through the soil is quite complex. In RI systems the potential exists for a significant level of phosphorus to remain in solution after passing through the soil at the treatment site. The reaction in the soil system is primarily one of chemical sorption and precipitation. A mathematical model has been developed which permits predicting phosphate transport and transformation for an undirectional flow path. This model reported elsewhere (10, 11, 12), takes into consideration which compounds are formed and how fast they form. In addition, the amount of adsorption is estimated.

The design engineer will not ordinarily need to precisely estimate the phosphorus accumulation in the soil profile or the concentration in the soil solution. It is possible to make a "worst-case" estimate by assuming a neutral pH and estimating the detention in the soil profile using the equation:

$$kt = \ln \frac{C_o}{C_x} \qquad (5)$$

where k = instantaneous rate coefficient (0.002 hr^{-1})

t = detention time = $\frac{X\,\Theta}{\bar{V}}$

where X = distance along flow path (cm)
Θ = volumetric water content (cc/cc)
$\bar{V}$ = infiltration rate during system operation (cm/hr)

C_o = total phosphorus concentration in applied wastewater (mg/l)

C_x = total phosphorus concentration at point X along the flow path (mg/l).

Using this abbreviated approach does not allow one to predict the ultimate capacity of a soil. As the chemical composition of the soil changes, its capacity to react with phosphorus changes. At this time, it is not possible to obtain a definitive answer as to total capacity of the soil to react with phosphorus. Since phosphorus is not considered toxic and systems have been in operation for several decades without significant problems, this shortcoming is not considered serious.

RATIONALE EVALUATION

Many of the important parameters in the design of a rapid infiltration system are included in this design rationale. The design yields maximum loadings to achieve a predetermined level of treatment. The management of a system will drastically affect its treatment efficiency. Therefore, adequate design is no guarantee of adequate performance. Several additional factors must be included when making a complete design. The designer should also consider:

1) preapplication treatment
2) storage requirements
3) metal limitations

Several procedures can be used to evaluate the adequacy of a design rationale. The qualitative "how well" a system works depends on who is evaluating the system and the objectives of the treatment process. In each of the following examples, a system was designed based on conditions at the existing system. The design was then compared to system operation for an evaluation of the rationale. The first consideration was to determine, based on a design for hydraulic acceptability, whether the system should be capable

of accepting the loading reported. After establishing the hydraulic acceptability, a best estimate for the design loading to achieve required nitrogen treatment was developed based on the concentration of nitrogen reaching the water table. This is the same as determining the design hydraulic loading assuming the discharge permit for nitrogen is the same as the concentration observed at the ground water surface. In most cases where the treated effluent reaching the ground water does not meet drinking water standards for nitrogen, the system was reevaluated projecting modifications in system operation required to achieve adequate treatment.

Since phosphorus is not a critical element, the approximation procedure presented projects "worst-case" operation. Thus, we would anticipate observed phosphorus concentrations to be less than or equal to the projections.

Hollister, California

The data required for a complete design are presented in Table 5.

Determination of Maximum Acceptable Hydraulic Load--

a) determine SAR

$$SAR = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}}$$

$$SAR = \frac{11.4}{\sqrt{\frac{1.35 + 2.63}{2}}}$$

SAR = 8.1, therefore, not limiting.

b) determine limitation based on worst case hydraulic conductivity

$$6.4 \frac{cm}{hr} \times 24 \frac{hr}{day} \times 0.1 = 15.36 \text{ cm/day}$$

c) BOD limitation for hydraulic loading

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{220 \text{ mg/1000 cc}} = 10.23 \text{ cm/day}$$

d) SS limitation for hydraulic loading

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{274 \text{ mg/1000 cc}} = 8.21 \text{ cm/day}$$

Based on these calculations one would project the maximum hydraulic load of 8 cm/day. This is approximately twice the loading applied at the Hollister site.

TABLE 5. INPUT DATA REQUIRED TO EVALUATE HOLLISTER SITE (2)

Ca applied	54 mg/l	=	1.35 meq/l
Mg applied	64 mg/l	=	2.6 meq/l
Na applied	262 mg/l	=	11.4 meq/l
Hydraulic Conductivity of Restrictive Layer			6.4 cm/hr
BOD applied			220 mg/l
SS applied			274 mg/l
Temperature mean air			$18^{o}C$
TKN applied			39.8 mg/l
TKN at ground water (design TKN)			1 mg/l
NO_3^- applied			0.4 mg/l
NO_3^- at ground water (design NO_3^-)			0.6 mg/l
TOC applied			248 mg/l
Total Phosphorus applied			12.4 mg/l
Total Phosphorus at 7.7 m (design P)			8 mg/l

Estimate Hydraulic Loading to Achieve Water Quality Arriving at the Ground Water Interface 7-10 m (23-33 ft) Below the Surface for the Hollister Site--

a) Determine, based on carbon to nitrogen ratio, the maximum nitrogen removal from Equation 3

$$\Delta N = \frac{TOC-5}{2} = \frac{248-5}{2} = \frac{243}{2} = 121 \text{ mg/l}$$

Based on this determination, there is sufficient carbon to reduce all nitrogen forms to 1 mg/l or less.

b) Determine loading rate to obtain required level of nitrification from Equation 2

$$\text{Max daily loading} = \frac{30\ (T-3)}{X}$$

Since the TKN remaining after treatment is 1.1 mg/l,

$$\frac{30\ (18-3)}{39.8 - 1.1} = 11.6 \text{ cm/day}$$

the max loading to achieve nitrification is 11.6 cm/day.

c) Determine loading rate to obtain required level of denitrification from Equation 4

$$\text{Max daily loading} = \frac{40\ (T-5)}{Y}$$

Since the nitrate remaining after treatment is 0.6 mg/l,

$$\frac{40\ (18 - 5)}{38.7 + 0.4 - 0.6} = 13.5 \text{ cm/day}$$

The maximum design loading to achieve denitrification is 13.5 cm/day versus the 4 cm/day actually applied to the system. As stated earlier, concentrations of less than 1 mg/l are difficult to maintain.

Estimate Phosphorus Arriving at Ground Water Under Actual Application Conditions--

From a previous study (12), the water content and infiltration rate during system operation were estimates at 0.275 and 2.64 cm/hr, respectively. To calculate the maximum estimated phosphorus at the ground water surface 735 cm (24 ft) below the surface, one uses Equation 5

$$0.002 \text{ hr}^{-1} \frac{735 \text{ cm } (0.275)}{2.64 \text{ cm/hr}} = \ln \frac{12.4 \text{ mg/l}}{C_x}$$

where C_x is the solution concentration at 735 cm (24 ft). C_x in then equal to 10 mg/l. This is slightly greater than the 8 mg/l reported. For more precise estimates using a computer model, see Enfield et al, (11, 12).

Table 6 presents design loadings and concentrations versus reported values. For Hollister, the design compares favorably with practice.

TABLE 6. DESIGN EVALUATION FOR HOLLISTER, CA (2)

Parameter	Design	Reported
Hydraulic Acceptance cm/day	8	4
Max Loading for N removal * cm/day	11.6	4
BOD mg/l	≤ 5	6
SS mg/l	≤ 5	N.R. **
Phosphorus mg/l	10	8

* Concentrations less than 1 mg/l not consistently obtainable thru RI. Limitation is based on ability to nitrify.

** N.R. is not reported.

Milton, Wisconsin

The data required for a complete design are presented in Table 7.

Determination of Maximum Acceptable Hydraulic Load--

a) Determine SAR

$$SAR = \frac{Na}{\sqrt{\frac{Ca + Na}{2}}} = \frac{6.4}{\sqrt{\frac{1.8 + 1.5}{2}}} = 5$$

SAR is, therefore, not limiting.

b) Determine limitation based on worst case hydraulic conductivity. The vertical hydraulic conductivity was not measured at this site. It was estimated as 25 percent of the horizontal permeability or approximately 50 cm/hr. Then

50 cm/hr x 24 hr/day x 0.1 = 120 cm/day

TABLE 7. INPUT DATA REQUIRED TO EVALUATE MILTON SITE (5, pg. 47,48,57)

Ca applied	70 mg/l	=	1.8 meq/l
Mg applied	37 mg/l	=	1.5 meq/l
Na applied	147 mg/l	=	6.4 meq/l
Hydraulic Conductivity of restrictive layer*			50 cm/hr
BOD applied			28 mg/l
SS applied			29 mg/l
Temperature mean air			11°C
TKN applied			25.5 mg/l
TKN at ground water (design TKN)			21.0 mg/l
NO_3^- applied			5.3 mg/l
NO_3^- at ground water (design NO_3^-)			0.8 mg/l
TOC applied			36 mg/l
Total Phosphorus applied			8.2 mg/l
Total Phosphorus at 8 m (design P)			5.92 mg/l

* Estimated from horizontal permeability.

c) BOD limitation for hydraulic loading

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{28 \text{ mg/1000 cc}} = 80 \text{ cm/day}$$

d) Suspended solids limitation for hydraulic loading

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{29 \text{ mg/1000 cc}} = 78 \text{ cm/day}$$

Based on these calculations, one would project the maximum hydraulic load of 78 cm/day. This is approximately 20 percent greater than actual practice at the Milton site.

Estimate Hydraulic Loading to Achieve Water Quality Arriving at the Ground Water Interface 8 m (26 ft) Below the Surface for the Milton Site--

a) Determine, based on carbon to nitrogen ratio, the maximum nitrogen removal possible from Equation 3

$$\Delta N = \frac{TOC-5}{2} = \frac{36-5}{2} = 15.5 \text{ mg/l}$$

Based on this calculation, there is not sufficient carbon to reduce the total nitrogen below 15.3 mg/l (the total nitrogen reported in the vicinity of site is 21.8 mg/l). Regardless of management practice, this system should not be capable of meeting drinking water standards for nitrogen, since ammonia and organic nitrogen can be rapidly converted to nitrate in the presence of oxygen.

b) Determine loading rate to obtain required level of nitrification from Equation 2. Since the TKN remaining after treatment is 21.0 mg/l,

$$\text{Max daily loading} = \frac{30\ (T-3)}{X} = \frac{30\ (11-3)}{25.5 - 21.0} = 53 \text{ cm/day}$$

The design maximum loading rate to achieve nitrification is 53 cm/day.

c) Determine loading rate to obtain required level of denitrification from Equation 4. Since the nitrate remaining after treatment is 0.8 mg/l,

$$\text{Max loading} = \frac{40\ (T-5)}{Y} = \frac{40\ (11-5)}{4.5 + 5.3 - 0.8} = 27 \text{ cm/day}$$

The design maximum loading to achieve denitrification is 27 cm/day. The design rationale is conservative particularly in the area of denitrification. The rate of denitrification appears to be significantly greater than projected by Equation 4.

Estimate Phosphorus Arriving at Ground Water Under Actual Application Conditions--

The water content and application rate are estimated at 0.3 and 2.8 cm/hr, respectively (2.8 cm/hr was selected which is in the range reported with continuous operation). Then estimating the phosphorus concentration at a depth of 800 cm (26 ft) using Equation 5

$$kt = \ln \frac{Co}{Cx}$$

$$0.002 \text{ hr}^{-1}\left(\frac{800 \text{ cm } (0.3)}{2.8 \text{ cm/hr}}\right) = \ln \frac{8.2 \text{ mg/l}}{C_x}$$

one sees the estimated phosphorus concentration is 7 mg/l.

Table 8 presents design loadings and concentrations versus reported values. The observed nutrients are consistantly lower in concentration than those projected from the design rationale.

TABLE 8. DESIGN EVALUATION FOR MILTON, WISCONSIN

Parameter	Design	Reported
Hydraulic acceptance cm/day	78	67
Max loading for N removal cm/day	27*, 53**	67
BOD mg/l	$\geq$ 5	5.2
SS mg/l	$\geq$ 5	N. R.
Phosphorus at 8 m (26 ft.) mg/l	7	5.92

* Based on limitation for denitrification

** Based on limitation for nitrification
*** N. R. is not reported

Lake George, New York

The data required for a complete design is presented in Table 9.

Determination of Maximum Acceptable Hydraulic Load--

a) Determine SAR

TABLE 9. INPUT DATA REQUIRED TO EVALUATE LAKE GEORGE SITE (3)

Ca applied	18 mg/l	=	0.4 meq/l
Mg applied	5 mg/l	=	0.2 meq/l
Na applied	13 mg/l	=	0.6 meq/l
Hydraulic Conductivity of restrictive layer			2.9 cm/hr
BOD applied			28 mg/l
SS applied			N. R.**
Temperature as monitored in ground water			11°C
TKN applied			19.6 mg/l
TKN at ground water (design TKN)			0.3 mg/l
NO_3^- applied			3.2 mg/l
NO_3^- at ground water (design NO_3^-)			3.6 mg/l
TOC applied			42 mg/l*
Total Phosphorus applied			4 mg/l
Total Phosphorus at 22 m (72 ft.)			0.3-1.1

* Estimated TOC ≈ 1.5 x BOD

** N. R. is not reported

$$SAR = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}} = \frac{0.6}{\sqrt{\frac{0.4 + 0.2}{2}}} = 1$$

SAR is, therefore, not limiting.

b) Determine limitation based on the worst case of hydraulic conductivity.

2.9 cm/hr x 24 hr/day x 0.1 = 7 cm/day

c) BOD limitation for hydraulic loading is

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{28 \text{ mg/1000 cc}} = 80 \text{ cm/day}$$

d) Suspended solids limitation for hydraulic loading.

Suspended solids were not measured at the Lake George site. One would project a maximum hydraulic loading less than or equal to 7 cm/day based on the most restrictive layer approach. Apparently there is sufficient soil of a greater permeability to permit a higher hydraulic loading than estimated.

Estimated Hydraulic Loading to Achieve Water Quality Arriving at the Ground Water Interface 22 m (72 ft) Below the Surface for the Lake George Site--

a) Determine, based on carbon to nitrogen ratio, the maximum nitrogen removal possible from Equation 3.

$$\Delta N = \frac{TOC-5}{2} = \frac{42-5}{2} = 18.5 \text{ mg/l}$$

Based on this calculation, it is possible to reduce the total nitrogen from 25.8 mg/l to 7.3 mg/l with optimum management. Since the TOC is estimated from the BOD, the actual optimum level is subject to the reliability of estimates of TOC.

b) Determine loading rate, using Equation 2, to obtain required level of nitrification. Since the TKN remaining after treatment is 0.3 mg/l and the TKN applied was 19.6 mg/l, then

$$\text{Max daily loading} = \frac{30\ (T-3)}{X} = \frac{30\ (11-3)}{19.6 - 0.3} = 12 \text{ cm/day}$$

The design maximum loading to achieve nitrification is 12 cm/day which is slightly conservative based on the actual practice.

c) Determine loading rate, using Equation 4, to obtain required level of denitrification. Since the nitrate remaining after treatment is 3.6 mg/l,

$$\text{Max loading} = \frac{40\ (T-5)}{Y} = \frac{40\ (11-5)}{19.3 + 3.2 - 3.6} = 12.7\ \text{cm/day}$$

Estimate Phosphorus Arriving at Ground Water Under Actual Application Conditions--

The water content and application rate are estimated at 0.5 and 2 cm/hr, respectively. Then estimating the phosphorus concentration at a depth of 2200 cm (72 ft) using Equation 5,

$$kt = \ln \frac{Co}{Cx}$$

$$0.002\ \text{hr}^{-1}\left(\frac{2200\ \text{cm}\ (0.5)}{2\ \text{cm/hr}}\right) = \ln \frac{4}{C_x}$$

$$C_x = 1.3\ \text{mg/l}$$

one sees the phosphorus is estimated to be 1.3 mg/l compared to the 0.3 to 1.1 which is the range of values observed at well 11s. This well was sampled 22 m (72 ft) below the surface.

Table 10 presents design loadings and concentrations versus reported observations. The observed nutrients are essentially the same as those projected from the design rationale.

TABLE 10. DESIGN EVALUATION FOR LAKE GEORGE

Parameters	Design	Reported
Hydraulic acceptance * cm/day	7	13
Max loading for N removal ** cm/day	12	13
BOD mg/l	≤ 5	< 5
SS mg/l	≤ 5	N.R.***
Phosphorus at 22 m mg/l	1.3	0.3-1.1

* Based on hydraulic conductivity of most restrictive layer

** Based on limitations for nitrification

*** N.R. is not reported

Vineland, New Jersey

The data required for a complete design are presented in Table 11.

Determination of Maximum Acceptable Hydraulic Load--

a) Determine SAR

$$SAR = \frac{Na}{\sqrt{\frac{Ca + Mg}{2}}} = \frac{2.16}{\sqrt{\frac{0.3 + 0.16}{2}}} = 5.4$$

SAR is, therefore, not limiting since 9 is the value where the designer should become concerned.

b) Determine limitation based on worst case hydraulic conductivity.

15.8 cm/hr x 24 hr/day x 0.1 = 38 cm/day

c) BOD limitation for hydraulic loading

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{154 \text{ mg/1000 cc}} = 15 \text{ cm/day}$$

d) Suspended solids limitation for hydraulic loading

$$\frac{2.25 \text{ mg/cm}^2\text{-day}}{43 \text{ mg/1000 cc}} = 52 \text{ cm/day}$$

The limiting criteria for hydraulic loading is BOD with a maximum design loading of 15 cm/day.

Estimate Hydraulic Loading to Achieve Water Quality of the Vineland Site--

a) Determine, based on carbon to nitrogen ratio, the maximum nitrogen removal possible from Equation 3.

$$\Delta N = \frac{TOC-5}{2} = \frac{108-5}{2} = 52 \text{ mg/l}$$

Based on this calculation, it should be possible to reduce the total nitrogen to 1 mg/l with proper design and management.

b) Determine loading rate, using Equation 2, to obtain required level of nitrification. Since the TKN remaining after treatment is 22.2 mg/l and the TKN applied is 40 mg/l

$$\text{Max daily loading} = \frac{30\ (T-3)}{X} = \frac{30\ (13-3)}{40 - 22.2} = 17 \text{ cm/day}$$

TABLE 11. INPUT DATA REQUIRED TO EVALUATE VINELAND SITE (4)

Ca applied	12 mg/l	=	0.3 meq/l
Mg applied	4 mg/l	=	0.16 meq/l
Na applied	60 mg/l	=	2.61 meq/l
Hydraulic Conductivity of restrictive layer			15.8 cm/hr
BOD applied			154 mg/l
SS applied			43 mg/l
Temperature mean annual air			13°C
TKN applied			40 mg/l
TKN at ground water (design TKN)			22.2 mg/l
NO_3^- applied			-
NO_3^- at ground water (design NO_3^-)			1.5 mg/l
TOC applied			108 mg/l
Total soluble Phosphorus applied*			4.5-7.2 mg/l
Total Phosphorus at 150 cm*			4.3 mg/l

* (4, pg. 84)

The design maximum loading to achieve nitrification is 17 cm/day. This is significantly greater than the actual practice. There are two apparent problems at this site. First, the water table is mounding to within 90 cm (3 ft) of the soil surface. Thus, some sort of artificial drainage is required to achieve a minimum of 150 cm (5 ft) unsaturated profile at this location. The second problem appears to be with the continuous flooding. The operating schedule does not allow sufficient time for the system to oxidize and nitrify the wastewater. These are apparent management reasons for not achieving the design water quality.

c) Determine loading rate to obtain required level of denitrification. Since the nitrate remaining after treatment is 1.5 mg/l

$$\text{Max loading is } \frac{40\ (13-5)}{17.8 - 1.5} = 20 \text{ cm/day}$$

The design maximum loading to achieve denitrification is 20 cm/day. Again, there are management problems which apparently are limiting system operation.

Estimate Phosphorus Arriving at Ground Water Under Actual Application Conditions--

The water content and application are estimated at 0.5 and 0.5 cm/hr, respectively. The total soluble phosphorus applied at the basins ranged from 4.5 to 7.2 mg/l during the study period (4, pg. 84). But the mean total phosphorus was reported as 9.3 mg/l. At one well cased and perforated in the depth range of 120 to 180 cm (4 to 6 ft), the phosphorus ranged from 2.9 to 5.7 mg/l with a mean of 4.3 mg/l. Estimating the phosphorus concentration at a depth of 150 cm (5 ft) using Equation 5 gives

$$0.002 \text{ hr}^{-1}\left(\frac{150\ (0.5)}{0.5}\right) = \ln \frac{5.85}{C_x}$$

$$C_x = 4.3 \text{ mg/l}$$

which is the same as observed at the site.

Table 12 presents design loadings and concentrations versus reported values. If the design rationale is adequate, changing the operation of the system should improve the water quality reaching the ground water with respect to nitrogen. Phosphorus is being treated as expected.

CAPABILITY FOR IMPROVEMENT

The Vineland site should be capable of meeting primary drinking water standards by increasing the size of the plant, adding required drainage, and changing the scheduling of flooding cycles. Since sufficient carbon is available to denitrify the applied wastewater, it should be possible to design the system such that not more than 10 mg/l total nitrogen will reach the ground water. On the other hand, the Milton system could not be significantly

improved with present level of pretreatment, due to the lack of available carbon. Hollister and Lake George are operating their systems efficiently and no changes would be recommended based on this study.

TABLE 12. DESIGN EVALUATION FOR VINELAND

Parameters	Design	Reported
Hydraulic acceptance cm/day	15	3-6
Max loading for N removal* cm/day	17	3-6
BOD mg/l	$\leq$ 5	10
SS mg/l	$\leq$ 5	N. R.**
Phosphorus at 150 cm (5 ft.) mg/l	4.3	4.3

* Based on limitation due to nitrification.
** N. R. is not reported

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